The effect of vibratory pile driving on the original gravity structure quay wall at Ben Schoeman Dock Berth 603, Port of Cape Town

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INTRODUCTION

Due to the recent radical increase in container vessel sizes (Figure 1), ports around the world are being refurbished. The ports of Anchorage, Cape Town, Colombo, Durban, Miami, Mumbai and Rotterdam have all recently undergone, or are currently undergoing, expansion to accommodate larger container ships. Between 1914 and 1988, the largest container vessel in the world had a capacity of 5 000 TEU (twenty-foot equivalent units). Currently, the largest ships can carry 18 000 TEU. Ships of this capacity are 400 m long, 59 m wide and 73 m high. Such vessel sizes are manufactured to save costs on fuel, crews and multiple vessel deployments (Marcario 2013; Saltmarsh 2011).

The handling of container ship cargo requires that container ships be moored

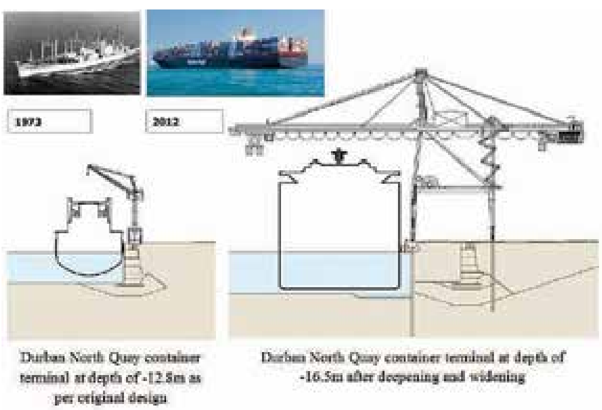


Figure 1: Illustration of increasing vessel and equipment sizes (Source: Transnet 2012)

in the protected and relatively calm waters of a port. However, the ships allowed to enter a given port are restricted due to the crane-height, under-keel clearance, infrastructure and equipment of the port – and there are only a few ports in the world able to accommodate the largest vessels (Marcario 2013). The increasing popularity of larger container ships in global trade prompted the expansion and refurbishment of the Cape Town container terminal (Figure 2).

In January 2007, the Cape Town container terminal (CTCT) expansion project (henceforth referred to as the *Project*) commenced. The Project finished in early 2013 and was undertaken to increase the container capacity of the Port of Cape Town from 0.8 million TEU to 1.4 million TEU per annum (Basson 2010). The container terminal consists of four operational berths: Berth 601 to Berth 604 (Figure 3). Prior to refurbishment, the original quay wall structure for all four berths was concrete blockwork – a form of gravity earth-retaining structure. In addition to reconfiguring the container terminal to maximise stack space, the Project involved the deepening of the Ben Schoeman basin and the installation of larger ship-to-shore (STS) cranes.

Figure 2: Plan of the Port of Cape Town (Source: Google Earth Data 2013)

Figure 3: Plan of the Ben Schoeman Dock container terminal (Source: Google Earth Data 2013)

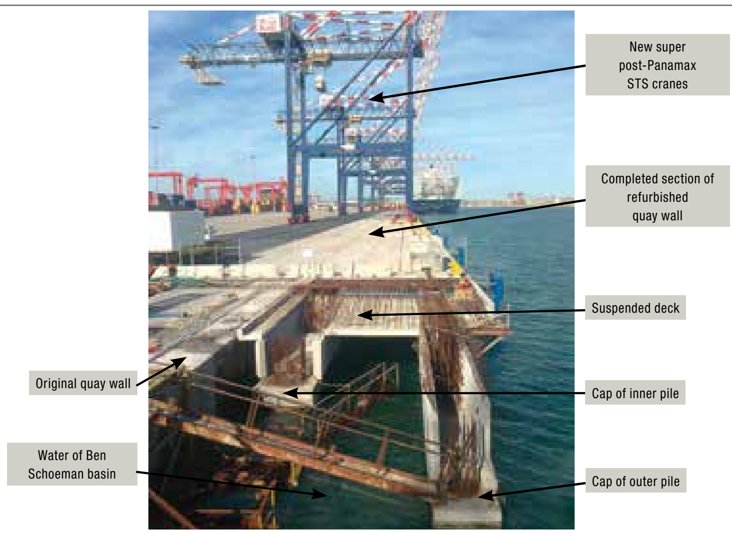


Figure 4: New suspended deck at the Cape Town container terminal Berth 603 (Source: Ben Schoeman Project June 2011)

PROBLEM STATEMENT

Geotechnical stability analyses conducted during the initial stages of the Project indicated that the deepening of the Ben Schoeman basin, in conjunction with increased horizontal and vertical loading due to larger STS cranes, would result in a significant decrease in the stability of the original quay wall (Mott MacDonald 2008). Hence, there was a need to design appropriate works that ensured stability of the refurbished quay wall structure in its final operating condition. For the stretch of quay wall along Berths 602 to 604, the chosen stability solution was to install two rows of bearing piles adjacent to the original quay wall structure on the seaward side (Figure 4). The bearing piles took the form of hollow, tubular, open-ended steel casings, driven to design depth using a vibratory pile hammer and filled in situ with reinforced tremie concrete. These piles served the dual purpose of stabilising the existing quay wall and supporting a new suspended deck. During quay refurbishment, various movements of the original quay wall were detected by the Project authorities. The movements were largely due to the creation of variably extensive voids beneath the toe of the existing quay, resulting from vibrational compaction of granular foundation materials during installation of the inner pile row.

AIMS AND OBJECTIVES

The primary objective of this study was to investigate how vibratory pile driving adjacent to the quay wall toe influenced the geotechnical stability of the structure. This was achieved through assembly and analysis of a two-dimensional plane strain numerical model of the original quay wall at Berth 603. This led to an additional objective, namely to assess whether the quay wall movements – a three-dimensional problem – could be accurately simulated and analysed using a simplified 2D plane strain model. All modelling was completed using PLAXIS 2D 2012, a finite element software package used for the analysis of deformation and stability in geotechnical engineering. The reasons for focusing this study on a particular length of the container terminal at Berth 603 (henceforth referred to as the Site) are outlined in the following section.

PROJECT DESCRIPTION

Characterisation of the Site

The structural and geologic composition of the whole container terminal was assessed to (a) select the Site location, and (b) determine the most conservative input parameters for the assembly of the numerical model. The Site represents the section of the Ben Schoeman container terminal assumed to be the most critical for geotechnical stability analysis of the original quay wall.

The cross-section of the original blockwork wall is uniform along the length of the operational quay. However, logs of boreholes drilled along the length of the terminal, both in front of and behind the quay wall face, show that the underlying geology varies erratically. The materials of the foundation geology are classified into two categories:

- Imported construction materials, such as sand fill, crushed stone and rock rubble, situated immediately below and behind the quay wall, and
- Lower-lying in situ material of the Tygerberg formation, weathered to various extents, ranging between hard rock and residual soil (a very stiff silt/clay).

The initial stages of the original quay wall construction in 1971 involved the excavation of in situ seabed material to create a trench running along the length of the terminal, which was then filled with granular construction materials to form the blockwork wall foundation bed. The cross-sectional geometry of this trench (perpendicular to the quay wall face) varies in depth and breadth along the length of the quay. Selection of the Site location was based primarily on determining the point along the quay where the response of the foundation

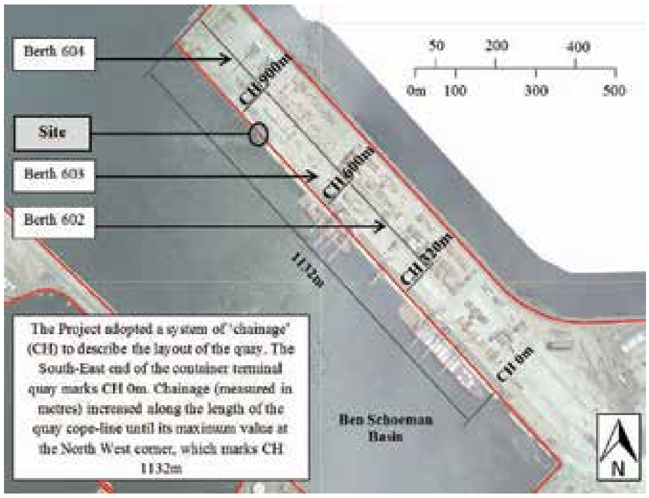


Figure 5: Location of Site along Ben Schoeman container terminal quay (Source: Wegener 2013)

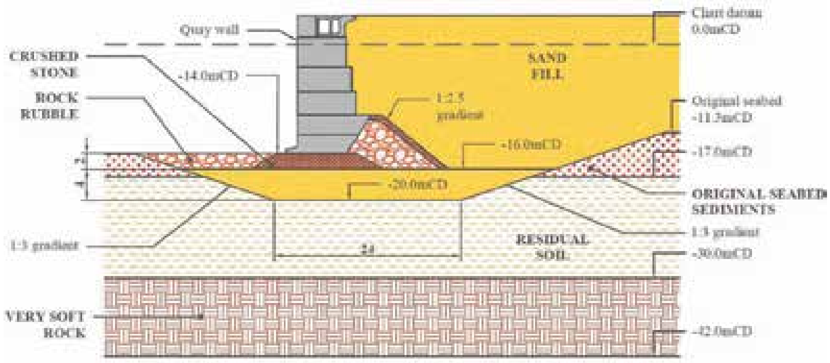


Figure 6: Site foundation profile (Source: Wegener 2013)

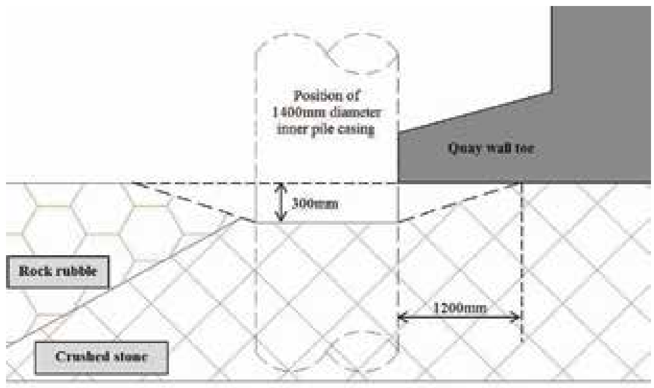


Figure 7: Approximate void size below quay wall toe at Site (Source: Wegener 2013)

Table 1: Summary of materials and their associated constitutive models

Material			Selected constitutive material model
Soil	In situ materials	Original sea bed sediments	Hardening soil with small-strain stiffness (HSsmall) model[1]
		Residual soil	Hardening soil (HS) model[2]
		Very soft rock	HS
	Construction materials	Sandfill	HSsmall
		Crushed stone	HS
		Rock rubble	HS
Mass concrete			Linear Elastic (LE)[3]/non-porous
1= (most) advanced model; 2 = advanced model; 3 = basic model			

conditions to vibratory pile driving had the most detrimental effect on quay wall stability. In doing so, it was assumed that:

- The depth and extent of the cones of settlement formed in the foundation material around the driven piles increased as the trench increased in thickness and breadth (i.e. as the thickness of granular materials, prone to immediate settlement by vibrational compaction, increased), and
- Wall stability analysis was more sensitive to the material properties of the imported construction materials than to the strength of the underlying in situ material.

Bearing this in mind, and through review of borehole logs, the Site location was selected to be the length of quay-spanning chainage (CH) 845 m–855 m (Figure 5). Here, the foundation profile has the widest trench, deepest trench invert level, thickest sandfill layer, and thickest crushed stone foundation bed. The critical foundation profile, as well the blockwork wall, is illustrated in Figure 6.

For the length of quay at the Site, the cones of settlement around the driven piles were, according to dive inspection reports compiled by Project authorities, approximately 300 mm deep and extended roughly 1 200 mm from the pile casing wall (Figure 7).

Once the geometry of the quay wall and its foundation conditions had been determined, it was necessary to derive material parameters for both the wall and soil for incorporation into the PLAXIS 2D model.

Material parameter selection

Table 1 summarises the constitutive relationships selected to model each of the materials within the numerical model. Constitutive model selection was based on recommendations made in Plaxis (2012) regarding the behaviour of various material types under certain loading conditions. While more basic models were chosen where possible, it was realised that

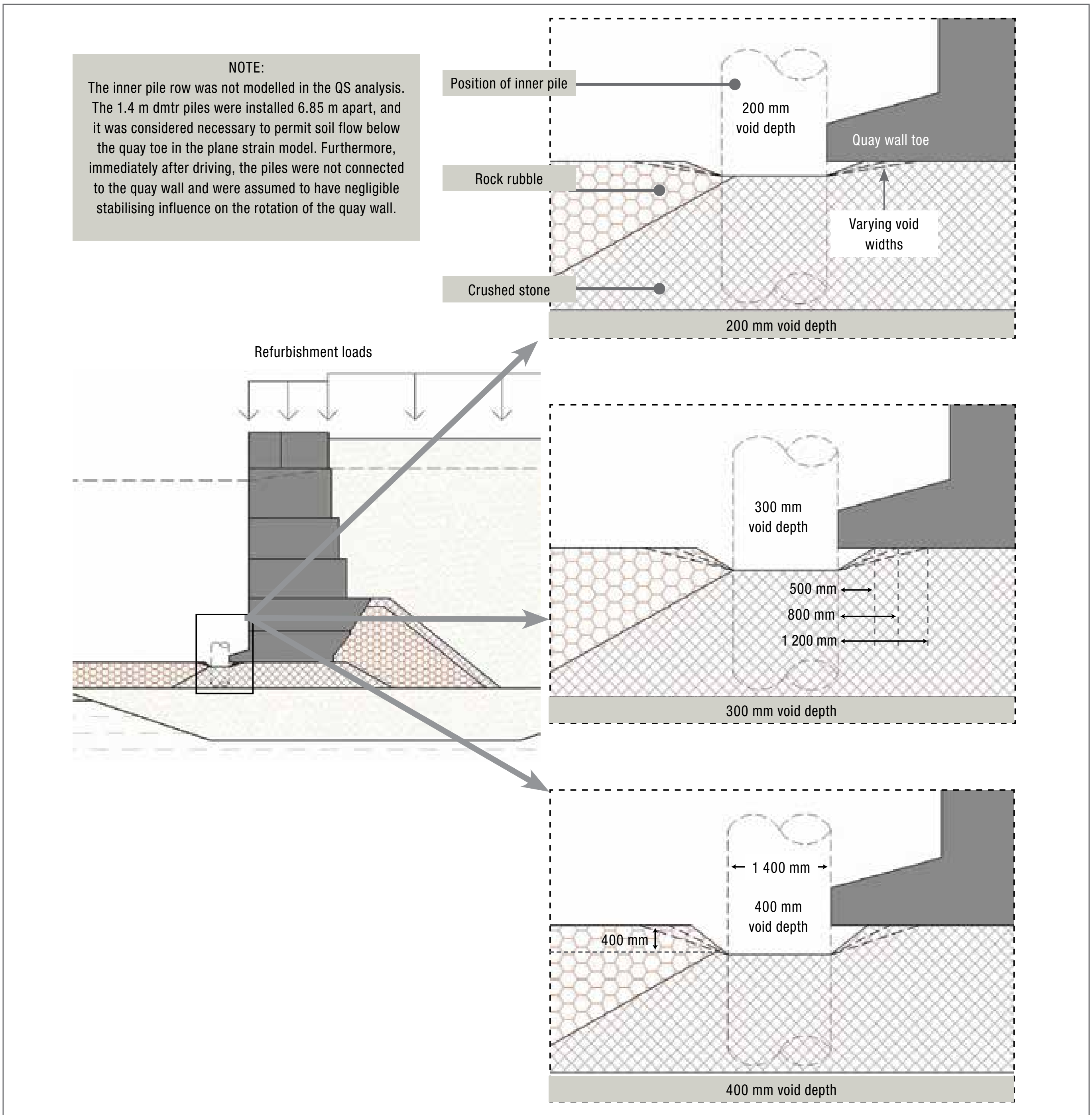


Figure 8: Void sizes modelled for the quasi-static analysis (Source: Wegener 2013)

The model analysis was not time-dependent and the results shown in Figure 10 represent the instantaneous cumulative increase in horizontal deflection of the quay wall as a result of void formation. The model results are comparable to the total cumulative deflection shown in Figure 9 (45 mm).

more advanced models – although associated with more complex parameter derivation and computation time – yielded more accurate results.

Selection of soil parameters was based on (a) consultation with working engineers (Holmwood 2013), (b) engineering judgement based on the mineralogy and fabric of the various foundation materials, and (c) literature-derived estimations using raw geotechnical test data specified in Project reports – with special reference to Brinkgreave *et al* (2010), Lees (2012), Plaxis (2012), and ZLH (2009).

All soil parameters were derived for drained conditions; the reason for doing so is made clear in the following section. A full list of the selected soil parameters is available upon request from the first author.

Model assembly

The PLAXIS 2D 2012 software (henceforth referred to as *Plaxis*) allows finite element calculations to be divided into sequential calculation phases (or stages). Except for the first stage in which the initial stress conditions are calculated, each phase is a continuation of the previous phase and corresponds to a construction stage or particular load case. Every stage has its own unique set of calculation results based on the specified input geometry, material parameters and imposed loading.

The assembled model consisted of ten phases, which, in order to account for the complete stress history of the foundation soil, modelled the initial construction, operation and refurbishment of the original quay wall up to the point of pile installation. The phases were calculated as a set

Point in time	Description
A	Pile installation
B	Toe bucket installation
C	Casting of suspended deck RC slab

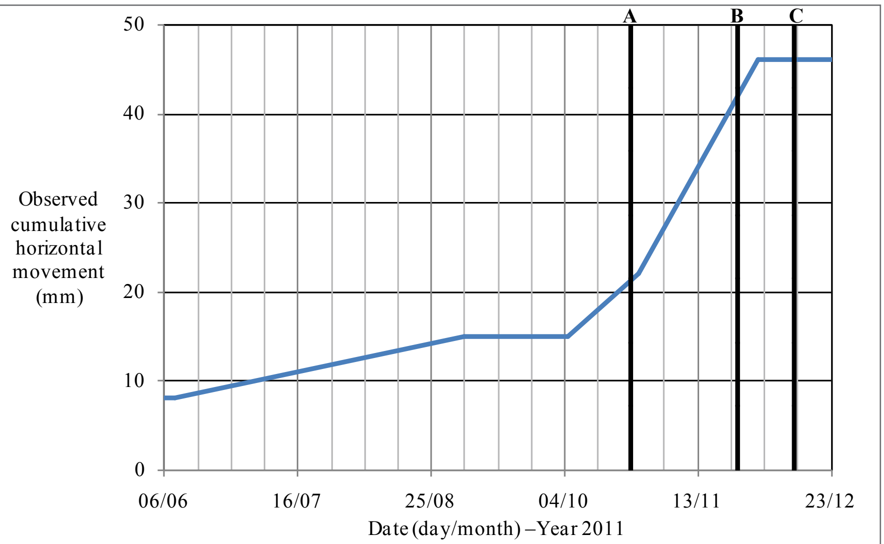
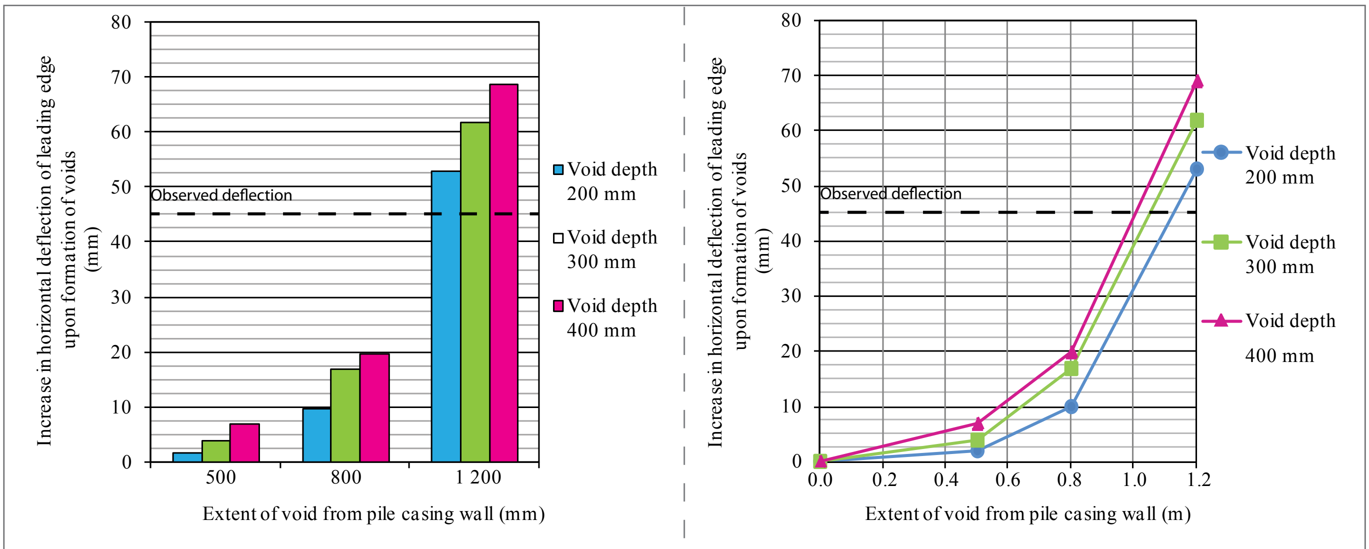


Figure 9: Observed horizontal movement of quay wall leading edge during refurbishment (Source: ZAA 2012)



a) Model results – grouped according to void width

b) Model results – presented on a scatter plot

Figure 10: Horizontal deflection of quay wall leading edge upon formation of voids (Source: Wegener 2013)

of sequential static analyses with instantaneous changes in loading and geometry between stages. However, in reality, there were considerable periods of time (no less than a few days) between each construction stage of the original quay wall – the operation phase would have lasted some 40 years. Hence, the assembly of the static model assumed drained soil conditions throughout all calculation phases for all soil types. The modelling of un-drained soil conditions throughout the phases would have yielded an unrealistic accumulation of excess pore water pressure in the lower-lying in situ, non-granular materials.

For the final calculation stage, a time-independent, quasi-static analysis was conducted to model the influence of the driven piles. The analysis considered the point in time just before the dynamic action of the piles was complete. To simulate the instantaneous state of the quay foundation soil, voids below the quay wall toe were incorporated into the model geometry. Due to difficulties experienced in using Plaxis to investigate the dynamic action of an embedded pile casing, the modelled void sizes were not based on any numerical results, but rather on on-site dive survey observations which described the cones of settlement formed around the pile centres. To cater for uncertainty surrounding the extent of settlement below the quay toe at the Site, a total of nine void sizes were modelled (Figure 8) – three depths and, for each depth, three different void widths.

RESULTS

The assessment of the model results consisted of (1) the comparison of the achieved deflections to the observed quay wall movement during construction at the site, and, once the accuracy of the model had been evaluated, (2) the assessment of the effect of the void formation on the stability of the original quay wall.

The cumulative horizontal deflection (Figure 9) of the Site quay wall leading edge observed during refurbishment was compared to the lateral deflections indicated by the numerical model output (Figure 10). It must be noted that the point of zero cumulative deflection was considered the position of the leading edge at the start of refurbishment, and that once the suspended deck was connected to the original wall, deflections ceased.

The model analysis was not time-dependent and the results shown in Figure 10 represent the instantaneous cumulative increase in horizontal deflection of the quay wall as a result of void formation. The model results are comparable to the total cumulative deflection shown in Figure 9 (45 mm).

Table 2: Results of ϕ/c reduction calculations for sub-models of the QS analysis

Void width (mm)	Void depth (mm)					
	200		300		400	
	Global FOS	Decrease in FOS from before driving	Global FOS	Decrease in FOS from before driving	Global FOS	Decrease in FOS from before driving
500	1.161	0.069	1.142	0.088	1.122	0.108
800	1.109	0.121	1.100	0.130	1.096	0.134
1 200	1.052	0.178	1.049	0.181	1.043	0.187

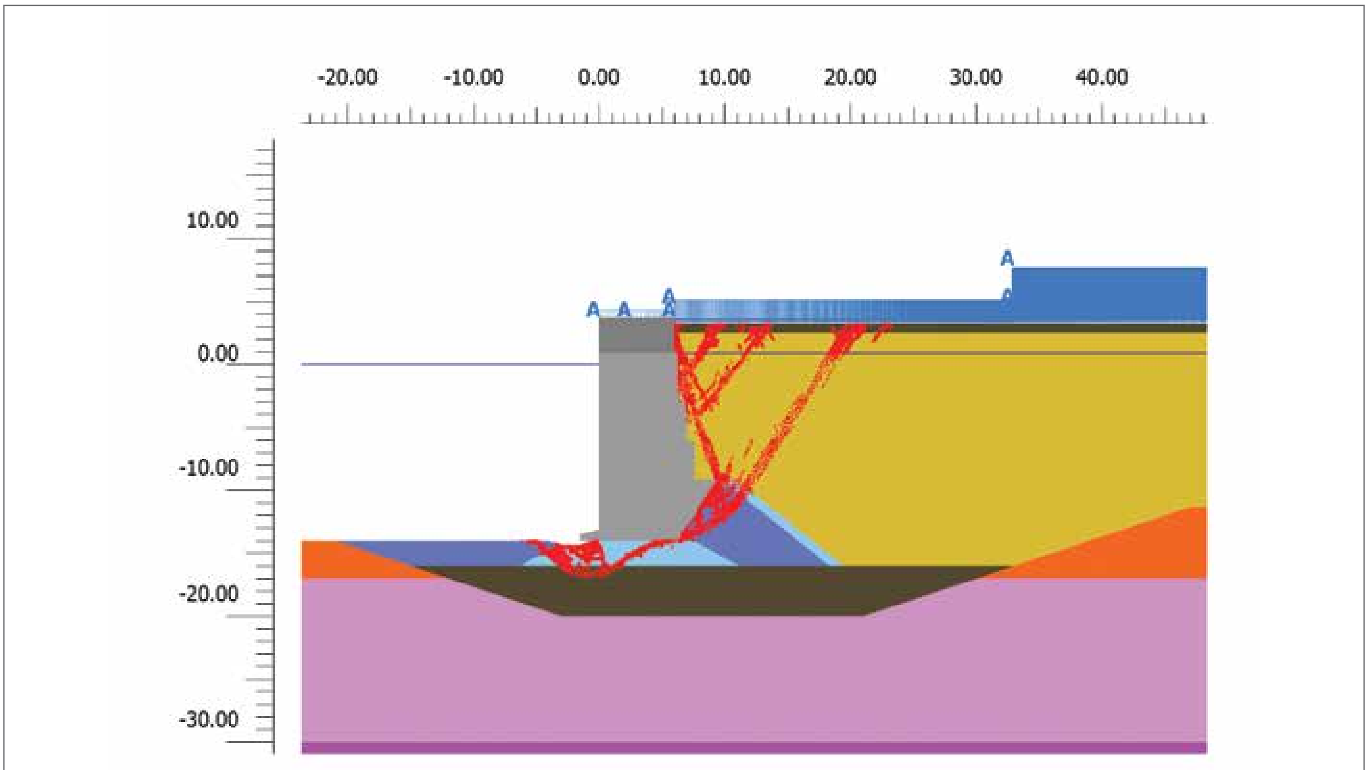


Figure 11: Location of failure points in soil. Quasi-static model safety analysis – Void: 1 200 mm wide, 400 mm deep

Modelling output confirmed that the original quay wall remained stable both before and immediately after installation of piles adjacent to the toe. Nonetheless, the decrease in global stability of the blockwork wall as a result of vibratory pile driving was significant. Quay wall stability was restored once construction of the suspended deck was complete.

Figure 10(b) shows that the void size corresponding to the observed deflection is less than that of the observed void sizes below the quay wall. However, this result was expected – a plane strain model (which is a conservative representation of the quay foundation conditions) was assumed. Hence, if the measured wall movements were to match the results given by the numerical model, it is likely the void size would be larger in reality than that modelled in the 2D analysis (ZAA 2011a). Comparison of the achieved deflection values to the surveyed data show that the two sets of results do not differ by an order of magnitude. Therefore, the model results were considered sufficiently accurate to warrant further assessment.

The effect of void formation on the stability of the quay wall was assessed through:

- Consideration of the global FOS of the wall, and
- The primary failure mechanism of the wall.

According to the penultimate model calculation phase, the quay wall had a factor of safety of 1.230 immediately prior to refurbishment. Model output indicated the extent of the decrease in stability of the original quay wall (Table 2) as a result of vibratory pile driving.

The output showed in Table 2 confirmed that the original quay wall remained stable throughout refurbishment. However, the decrease in stability due to void formation was significant, especially for voids of width 1 200 mm, where the global FOS values ranged between 1.043 and 1.052.

In assessing the safety of the quay wall, a cumulative horizontal deflection of 50 mm was considered the serviceability limit state (ZAA 2012). With this borne in mind, Figure 10 shows that the quay wall would have been deemed to fail due to excessive deformation before reaching a condition of collapse. Model results showed that the horizontal deflection corresponding to the largest modelled void was 69 mm – exceeding the specified deflection limit by 40%. However, the quay wall global FOS corresponding to the same void size is greater than 1.0.

The primary failure mechanisms for all modelled void sizes were the overturning of the quay wall and the formation of a shallow rotational slip surface. Other than local failure at the edge of the toe, there was no indication of bearing failure of the foundation soil. Sliding between the interface of the quay wall base and the foundation bed was absent. Figure 11 shows the location of failure points within the foundation soil and backfill of the model which incorporated the largest cross-sectional void area, i.e. 1 200 m wide and 400 mm deep.

CONCLUSIONS

The observed movement of the Ben Schoeman container terminal quay wall during its refurbishment was a three-dimensional problem – cones of settlement were localised around pile centres, the quay foundation profile varied erratically, and friction between adjacent blocks of the wall caused varying rates of deflection along the length of the quay. The assembled quasi-static numerical model of the critical quay wall section – although a conservative 2D representation of the problem – yielded results of acceptable accuracy. This was concluded through comparison of:

- The deflections indicated in the plane strain, quasi-static analysis results, and
- The observed on-site horizontal movements of the quay wall leading edge during completion of the Project.

Selection of the critical section of the quay wall for stability analysis was based primarily on assessment of the quay foundation profile – more specifically, the geometry and load-induced behaviour of the imported construction materials. Modelling output confirmed that the original quay wall remained stable both before and immediately after installation of piles adjacent to the toe. Nonetheless, the decrease in global stability of the blockwork wall as a result of vibratory pile driving was significant. Quay wall stability was restored once construction of the suspended deck was complete.

The cumulative horizontal deflections of the quay wall leading edge between the start of quay construction and the instant after pile installation were in the order of 210 mm; critical lateral movements subsequent to pile driving – but prior to suspended deck completion – were approximately 45 mm. Model results indicated that the quay wall would fail due to excessive deflection before an ultimate limit state was reached.

REFERENCES

The list of references is available from the first author, or from the editor. ◻